CHAPTER 10

INTERPRETATION OF ROCK PROPERTIES

10.1. INTRODUCTION

The engineering behavior of most rock masses under loading is determined primarily by the discontinuities, fractures, joints, fissures, cracks, and planes of weakness. The intact blocks of rock between the discontinuities are usually sufficiently strong, except in the case of weak & porous rocks and those that weather rapidly. Thus, two classification systems are needed to adequately characterize these geomaterials: one for the intact solid rock and another for the rock mass. The network of fractures divide the rock mass into discrete and prismatic blocks that affect its response and performance. With the exception of the durability testing (discussed in Chapter 8), the results of laboratory testing are of limited direct applicability to design of structures founded in or on rock masses.

Of the three primary rock types (igneous, metamorphic and sedimentary), sedimentary rocks comprise 75% of the rocks exposed at the ground surface. Among the sedimentary rocks, the rocks of the shale family (clay shale, siltstone, mudstone, and claystone) predominate, representing over 50% of the exposed sedimentary rocks worldwide (Foster, 1975). The distribution of rock types within the U.S.A. is reviewed by Witczak (1972) and Figure 10-1 shows a simplified map of their occurrence (Pough, 1988).

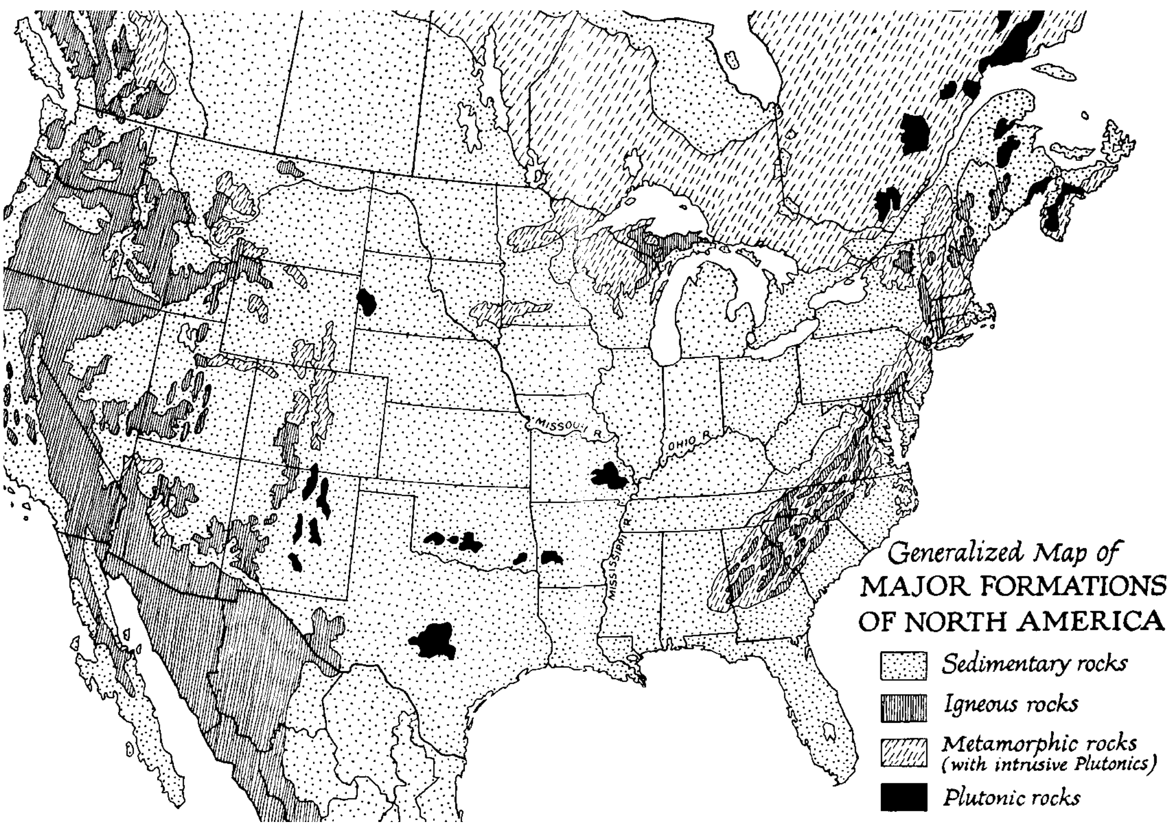

Figure 10-1. Generalized Distribution of Sedimentary, Igneous, & Metamorphic Rocks in the U.S.A
(From Pough, 1988)

An initial step during site reconnaissance and exploration is to categorize the basic type of rock, per Table 10-1. Detailed geological classifications of rock types and petrographic examinations in the laboratory will be required for major projects involving construction on rocks. Field mapping by engineering geologists is necessary for description of the jointing patterns, major discontinuity sets, shear zones, and faults, particularly in areas involving rock slopes, cliffs, tunnels, and bridge abutments. A detailed discussion of these aspects may be found elsewhere (e.g., Goodman, 1989; Pough, 1988). Major slip planes and joints should be detailed on maps with appropriate values of dip angle and dip direction (or alternatively, strike). Large groups of discontinuities are best represented by statistical summaries on stereonets and polar diagrams. Important shear zones and faults can also be depicted on these plots.

TABLE 10-1.

PRIMARY ROCK TYPES CLASSIFIED BY GEOLOGIC ORIGIN

Grains Aspects	Sedimentary Types		Metamorphic Types		Igneous Types	
	Clastic	**Carbonate**	**Foliated**	**Massive**	**Intrusive**	**Extrusive**
Coarse	*Conglomerate Breccia*	*Limestone Conglomerate*	*Gneiss*	*Marble*	*Pegmatiite Granite*	*Volcanic Breccia*
Medium	*Sandstone Siltstone*	*Limestone Chalk*	*Schist Phyllite*	*Quartzite*	*Diorite Diabase*	*Tuff*
Fine	*Shale Mudstone*	*Calcareous Mudstone*	*Slate*	*Amphibolite*	*Rhyolite*	*Basalt Obsidian*

Alternate classification systems are proposed based on behavioral aspects (Goodman, 1989) or composition and texture (Wyllie, 1999). Details on the specific rock minerals and their relative abundance is important in the petrographic determination of the rock types, yet beyond the scope of discussion here. In the logging of field mapping and rock coring operations, the specific formation name and age of the rock is often noted, being helpful in sorting stratigraphic layering and the determination of the subsurface profile. Table 10-2 gives the general geologic time scale and associated periods. Generally, older rocks have lower porosity and higher strength than younger rocks (Goodman, 1989).

Rock type can often infer possible problems that can be encountered in construction. Notable problems occur in limestone (sinkholes, caves), serpentine (slippage), bentonitic shales (swelling, slope stability), and diabase (boulders). Deterioration of shale family of rocks and weakly-cemented friable sandstones is the cause of many of the maintenance problems in the national highway system, particularly with respect to cuts, embankment construction, and foundations. For example, deterioration of cut slopes in shales will result in flatter slopes and/or instability. Shale used in embankments when compacted will break down and result in a material less pervious than anticipated for a rock fill. Maintenance problems for slopes can be mitigated by making them flatter, installation of horizontal drains, use of gunite & mesh, or in some cases, more elaborate structural supports are required (rock bolts, retention walls, anchors, drilled shafts). When excavation for a structural foundation is made, the bearing level must be protected against slaking and/or expansion; this can be accomplished by spraying a protective coating on the freshly exposed rock surface, such as gunite or shotcrete. Additional details and considerations are given in Wyllie (1999).

TABLE 10-2.

GEOLOGIC TIME SCALE

Era	*Period*	*Epoch*	*Time Boundaries (Years Ago)*
		Holocene - Recent	
	Quaternary		10,000
		Pleistocene	
			2 million
		Pliocene	
			5 million
Cenozoic		Miocene	
			26 million
	Tertiary	Oligocene	
			38 million
		Eocene	
			54 million
		Paleocene	
			65 million
	Cretaceous		
			130 million
Mesozoic	Jurassic		
			185 million
	Triassic		
			230 million
	Permian		
			265 million
		Pennsylvanian	
	Carboniferous		310 million
		Mississippian	
			355 million
Paleozoic	Devonian		
			413 million
	Silurian		
			425 million
	Ordovician		
			475 million
	Cambrian		
			570 million
Precambrian (oldest rocks)			3.9 billion
Earth Beginning			4.7 billion

The design of rock structures is still frequently done on the basis of an empirical evaluation of rock mass properties guided by experience, consideration of rock mass structure, index properties and correlations, and other parameters, such as joint spacing, roughness, degree of weathering, dip & dip direction of slip planes, infilling, extent of discontinuities, and groundwater conditions (see Figure 10-2). Many of these facets can be grouped together to give an overall rating of the predominant factors affecting the performance of the entire rock mass under loading. Thus, a rating of the rock mass will be described using three common methods (RMR = rock mass rating; Q system, and GSI = geologic strength index).

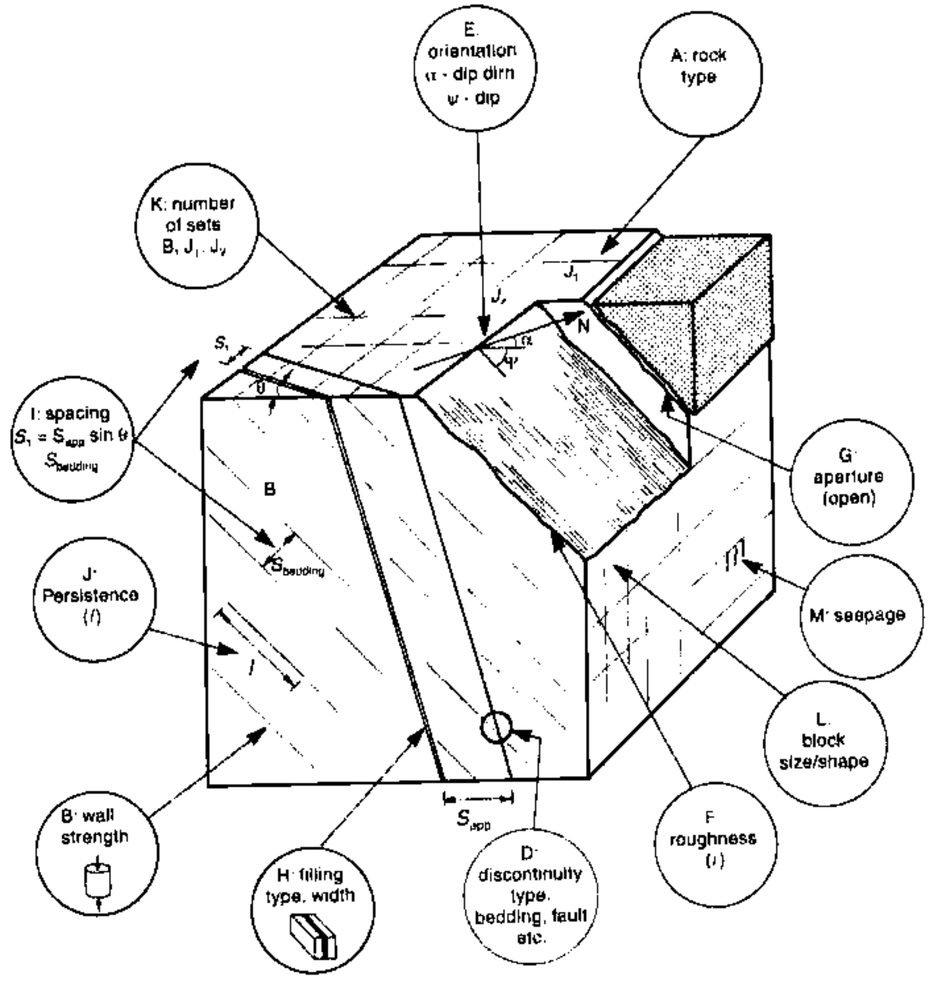

Figure 10-2. Factors & Parameters Affecting Geologic Mapping of Rock Mass Features (Wyllie, 1999)**.**

As in the case of the evaluation of soil properties, a number of correlations have been developed for the interpretation of rock properties. Notably, however, the rock property correlations reported in the technical literature often have a limited database and should be used with caution. An attempt should be made to develop correlations applicable to the specific rock formations in a particular state, as this can be well worth the expenditure of time and effort in terms of overall safety and economy.

This chapter presents general discussions on the properties of intact rock and jointed rock masses, particularly using rock mass classification schemes and their relevance to the design of rock structures. The reader is strongly encouraged to refer to the original references to understand the basis of the correlations and the classification systems presented in this chapter and for additional information.

10.2 INTACT ROCK PROPERTIES

This section presents information on the indices and properties of natural intact rock. The values are obtained from tests conducted in the laboratory on small specimens of rock and therefore must be adjusted to full scale conditions in order to represent the overall rock mass conditions.

10.2.1 Specific Gravity

The specific gravity of solids (G_s) of different rock types depends upon the minerals present and their relative percentage of composition. The values of G_s for selected minerals are presented in Figure 10-3. Very common minerals include quartz and feldspar, as well as calcite, chlorite, mica, and the clay mineral group (illite, kaolinite, smectite). The bulk value of these together gives an representative average value of G_s . 2.7 ± 0.1 for many rock types.

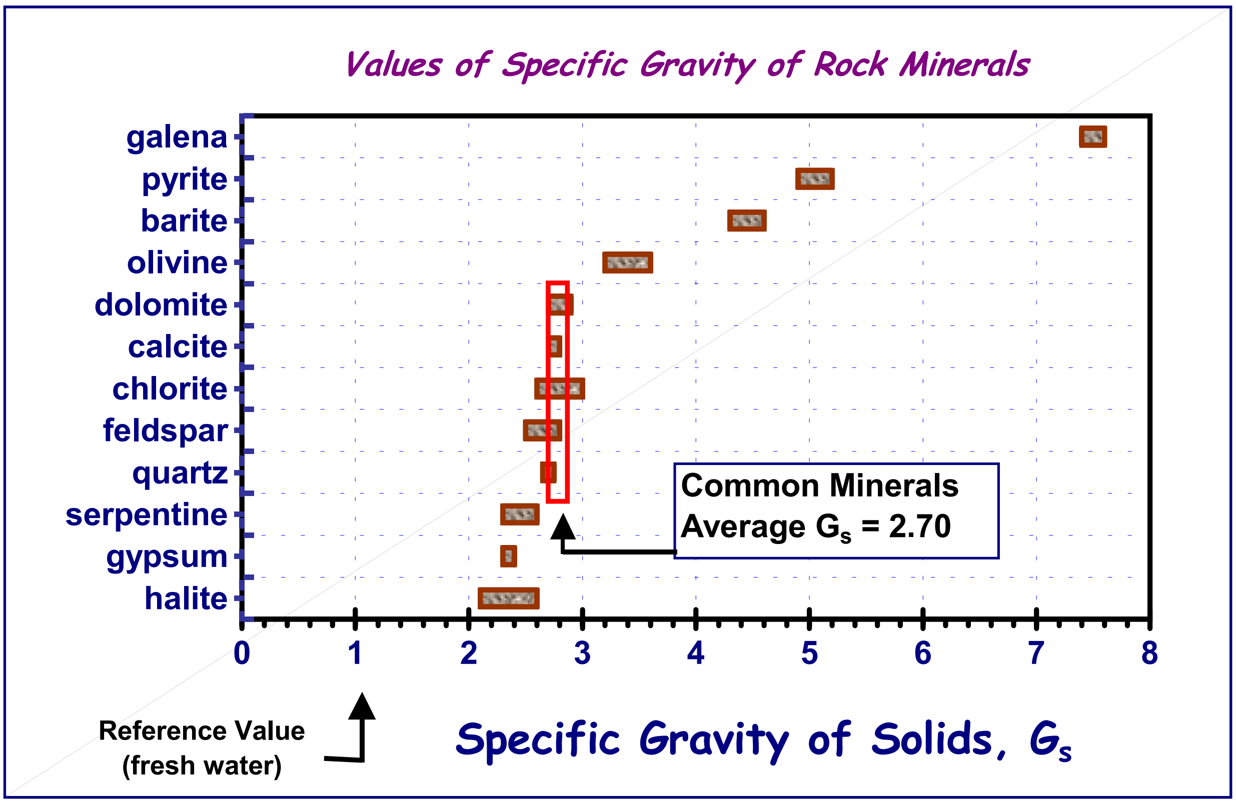

Figure 10-3. Specific Gravity of Solids for Selected Rock Minerals.

10.2.2. Unit Weight

The unit weight of rock is needed in calculating overburden stress profiles in problems involving rock slopes and tunnel design support systems. Also, because the specific gravity of the basic rock-forming minerals exhibits a narrow range, the unit weight is an indicator of the degree of induration of the rock unit and is thus an indirect indicator of rock strength. Strength of the intact rock material tends to increase proportionally to the increase in unit weight. Representative dry unit weights for different rock types are contained in Table 10-3.

TABLE 10-3

REPRESENTATIVE RANGE OF DRY UNIT WEIGHTS

Rock Type	Unit Weight Range (kN/m^3)
Shale	20 - 25
Sandstone	18 - 26
Limestone	19 - 27
Schist	23 - 28
Gneiss	23 - 29
Granite	25 - 29
Basalt	20 - 30

1. Dry unit weights are for moderately weathered to unweathered rock.. Note: 9.81 kN/m^3 = 62.4 pcf.
2. Wide range in unit weights for shale, sandstone, and limestone represents effect of variations in porosity, cementation, grain size, depth, and age.
3. Specimens with unit weights falling outside the ranges contained herein may be encountered.

The dry unit weight (γ_{dry}) is calculated from the bulk specific gravity of solids and porosity (n) according to:

$$\gamma_{dry} = \gamma_{water} G_s (1 - n) \qquad (10\text{-}1)$$

Where the unit weight of water is γ_{water} = 9.81 kN/m^3 = 62.43 pcf. The saturated unit weight (γ_{sat}) of rocks can be expressed:

$$\gamma_{sat} = \gamma_{water} [G_s (1 - n) + n] \qquad (10\text{-}2)$$

These expressions are consistent with those in Table 7-2 for soil materials where void ratio is used more commonly. The interrelationship between porosity and void ratio (e) is simply: $n = e/(1+e)$. The decrease in saturated unit weight with increasing porosity is presented in Figure 10-4 for various rocks and a selected range of specific gravity values.

10.2.3. Ultrasonic Velocities

The compression and shear wave velocities of rock specimens can be measured in the laboratory using ultrasonics techniques (see Section 8, Figure 8-7). These wave values can be used as indicators of the degree of weathering and soundness of the rock, as well as compared with in-situ field measurements that relate to the extent of fissuring and discontinuities of the larger rock mass. The summary of data in Figure 10-5 illustrates the general ranges of compression wave (V_p) between 3000 and 7000 m/s and ranges of shear waves (V_s) between 2000 and 3500 m/s for intact rocks.

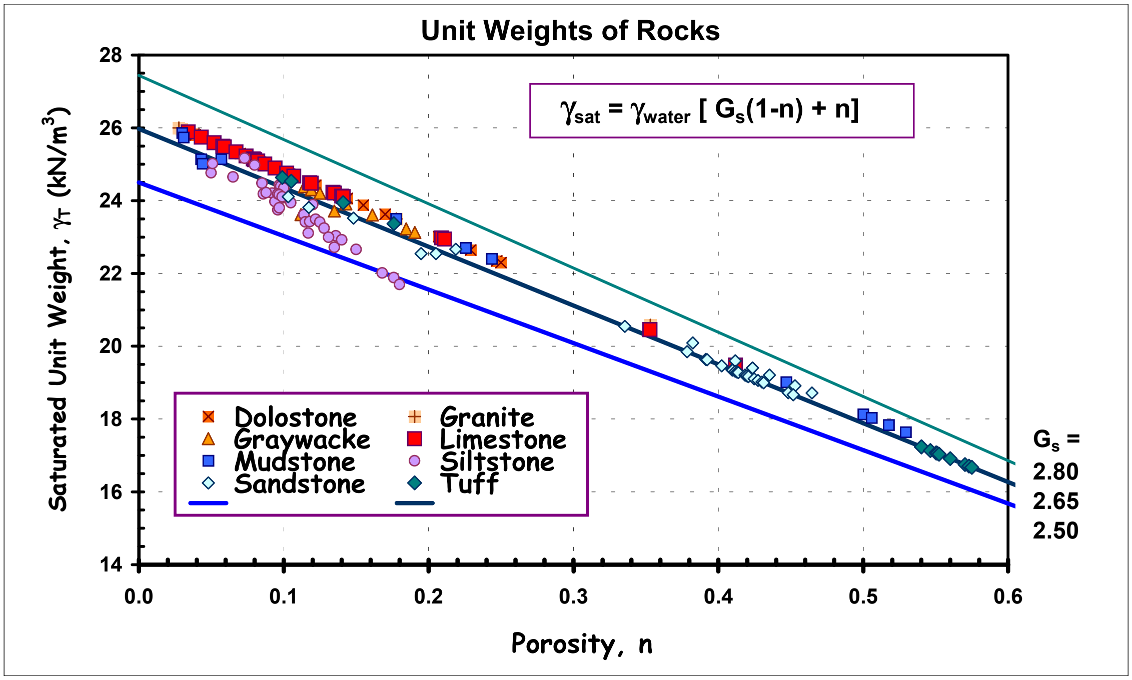

Figure 10-4. Saturated Rock Unit Weight in Terms of Porosity and Specific Gravity.

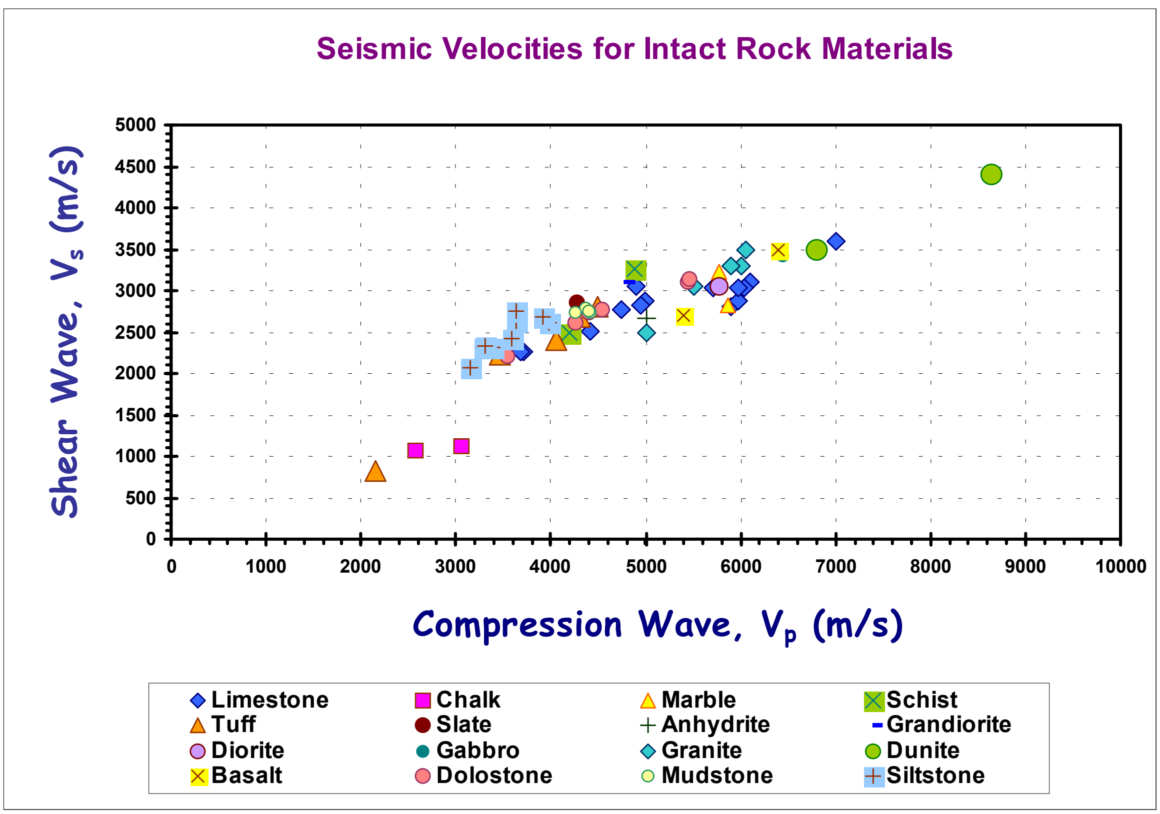

Figure 10-5. Representative S- and P-wave Velocities for Intact Rock Materials.

10.2.4 Compressive Strength

The stress-strain-strength behavior of intact rock specimens can be measured during a uniaxial compression test (unconfined compression), or the more elaborate triaxial test (See details in Figures 8-2 and 8-6). The peak stress of the F-, curve during unconfined loading is the uniaxial compressive strength (designated q_u or F_u). The value of q_u can be estimated from the point load index (I_s) that is easily conducted in the field (see Figure 8-1). Representative values of compressive strengths for a variety of intact rock specimens are listed in Table 10-4 (Goodman, 1989). For this database, the compressive strengths ranged from 11 to 355 MPa (1.6 to 51.5 ksi), with a mean value of q_u = 135 MPa (19.7 ksi). A wide range in compressive strength can exist for a particular geologic rock type, depending upon porosity, cementation, degree of weathering, formation heterogeneity, grain size angularity, and degree of interlocking of mineral grains. The compressive strength also depends upon the orientation of load application with respect to microstructure (e.g., foliation in metamorphic rocks and bedding planes in sedmentary rocks).

TABLE 10-4.
REPRESENTATIVE MEASURED PARAMETERS ON INTACT ROCK SPECIMENS
(modified after Goodman, 1989)

Intact Rock Material	q_u (MPa)	T_0 (MPa)	E_R (MPa)	ν (-)	Ratio q_u/T_0	Ratio E_R/q_u
Baraboo Quartzite	320.0	11.0	88320	0.11	29.1	276
Bedford Limestone	51.0	1.6	28509	0.29	32.3	559
Berea Sandstone	73.8	1.2	19262	0.38	63.0	261
Cedar City Tonalite	101.5	6.4	19184	0.17	15.9	189
Cherokee Marble	66.9	1.8	55795	0.25	37.4	834
Dworshak Dam Gneiss	162.0	6.9	53622	0.34	23.5	331
Flaming Gorge Shale	35.2	0.2	5526	0.25	167.6	157
Hackensack Siltstone	122.7	3.0	29571	0.22	41.5	241
John Day Basalt	355.0	14.5	83780	0.29	24.5	236
Lockport Dolomite	90.3	3.0	51020	0.34	29.8	565
Micaceous Shale	75.2	2.1	11130	0.29	36.3	148
Navajo Sandstone	214.0	8.1	39162	0.46	26.3	183
Nevada Basalt	148.0	13.1	34928	0.32	11.3	236
Nevada Granite	141.1	11.7	73795	0.22	12.1	523
Nevada Tuff	11.3	1.1	3649.9	0.29	10.0	323
Oneota Dolomite	86.9	4.4	43885	0.34	19.7	505
Palisades Diabase	241.0	11.4	81699	0.28	21.1	339
Pikes Peak Granite	226.0	11.9	70512	0.18	19.0	312
Quartz Mica Schist	55.2	0.5	20700	0.31	100.4	375
Solenhofen Limestone	245.0	4.0	63700	0.29	61.3	260
Taconic Marble	62.0	1.2	47926	0.40	53.0	773
Tavernalle Limestone	97.9	3.9	55803	0.30	25.0	570

Statistical Results:	Mean =	135.5	5.6	44613	0.29	39.1	372.5
	S.Dev. =	93.7	4.7	25716	0.08	35.6	193.8

Note: 1 MPa = 10.45 tsf = 145.1 psi

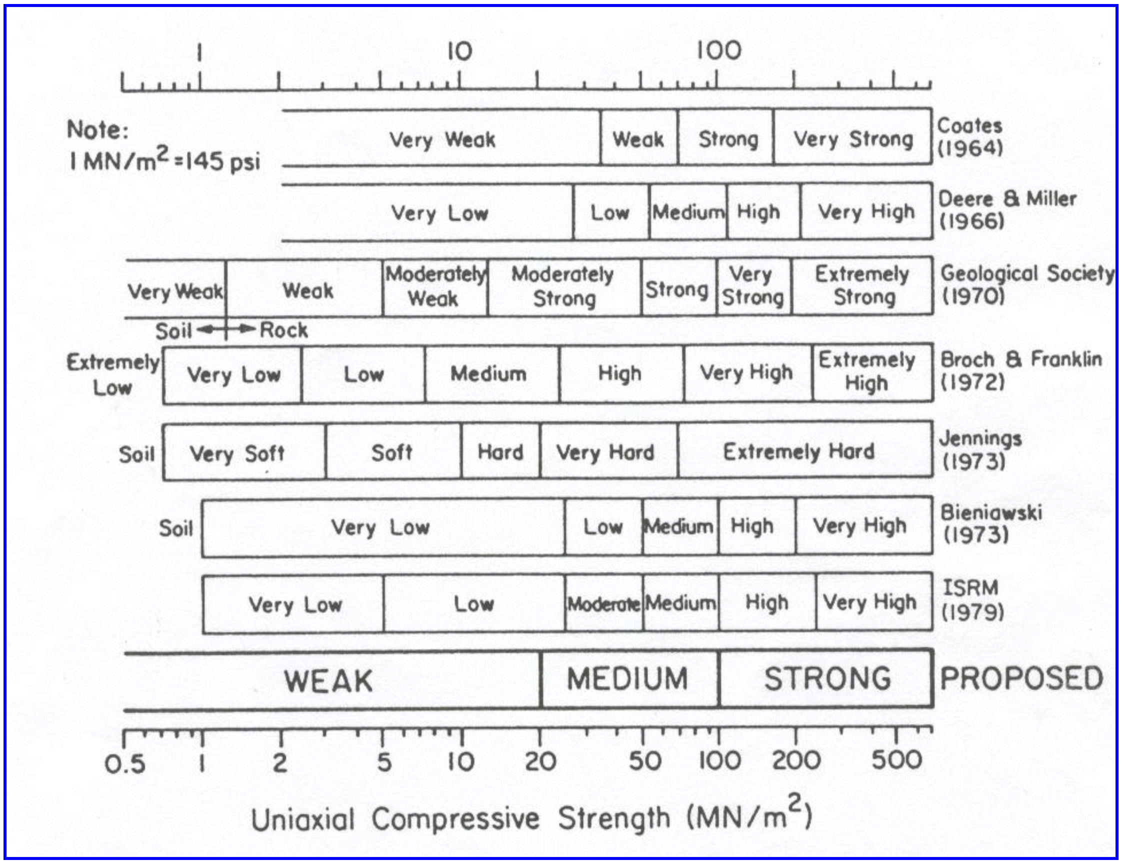

Figure 10-6. Classifications for Unweathered Intact Rock Material Strength
(Kulhawy, Trautmann, and O'Rourke, 1991)

The compressive strength serves as an initial index on the competency of intact rock. Figure 10-6 shows a comparison of several classification schemes. This is particularly useful for defining differences between hard clays to shales, as the boundary in the transition from soil to rock is not precise in these sedimentary materials. Similarly, it is applicable to residual profiles where the transition from soil to saprolite and weathered rock and rock may be needed. It can become important in contracts involving excavatability issues of rock vs. soil, as the former is considerably more expensive than the latter during site grading, deep excavations, and foundation construction.

10.2.5 Direct and Indirect Tensile Strength

Rock is relatively weak in tension, and thus, the tensile strength (T_0) of an intact rock is considerably less than its compressive value (q_u). Their interrelation in terms of Mohr strength criterion is shown in Figure 10-7. The direct tensile strength on rock specimens is not a common laboratory procedure because of the difficulties involved in proper end preparation (Jaeger and Cook, 1977). Therefore, it is usual to evaluate the tensile strength through indirect methods, including the split-tensile test (Brazilian test, per Figure 8-3), or alternatively, a bending test to obtain the modulus of rupture.

A list of representative tensile strength values for various rocks is given in Table 10-4 with a measured range from 0.2 to 14 MPa (30 to 2100 psi) and mean value T_0 = 5.6 MPa (812 psi). For the data considered, it can be seen from Figure 10-8 that the tensile strength averages only about 4% of the compressive strength for the same rock.

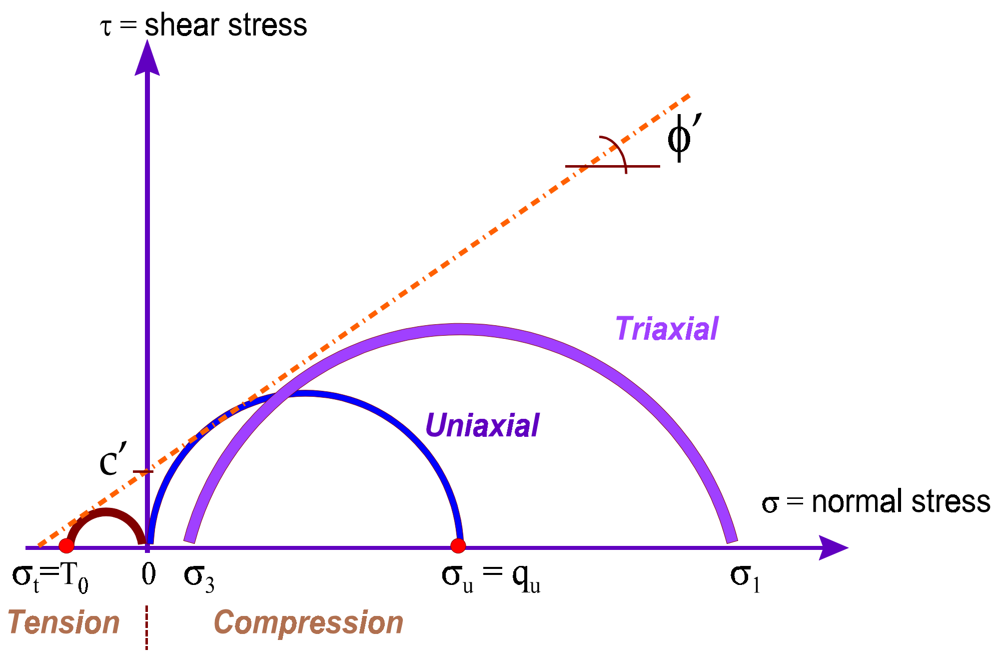

Figure 10-7. Interrelationship Between Uniaxial Compression, Triaxial, and Tensile Strength of Intact Rock in Mohr-Coulomb Diagram.

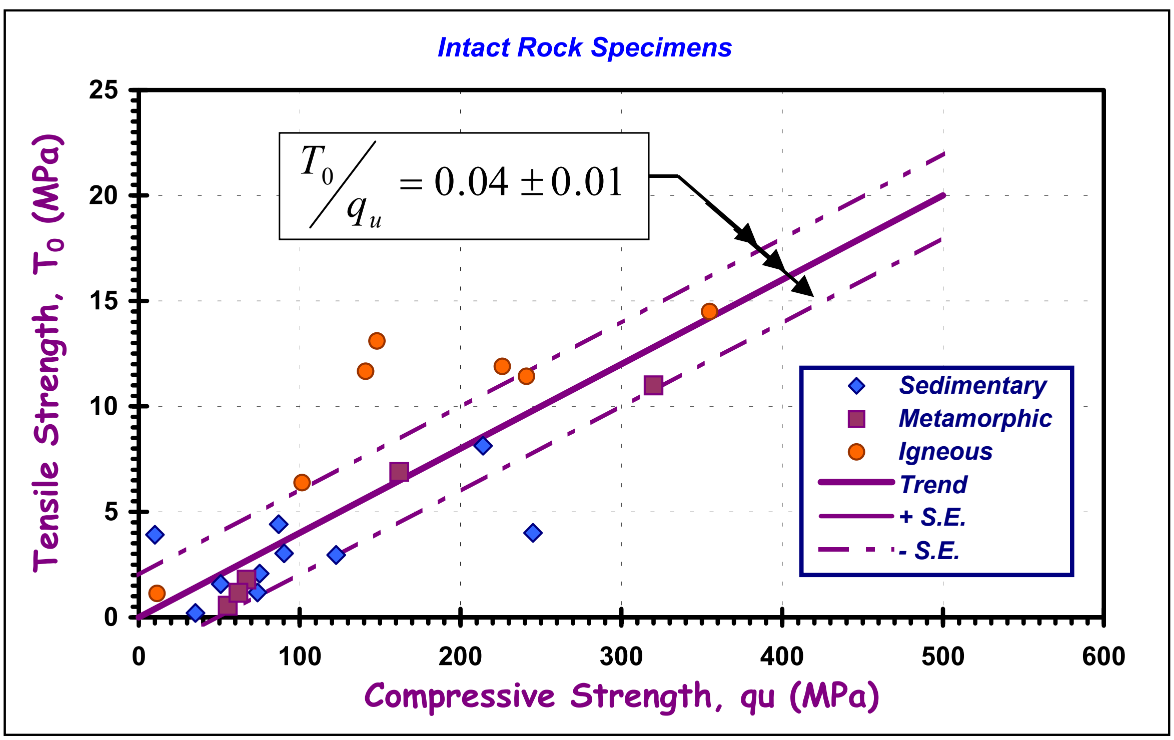

Figure 10-8. Comparison of Tensile vs. Compressive Strengths for Intact Rock Specimens.

10.2.6 Elastic Modulus of Intact Rock

The Young's modulus (E_R) of intact rock is measured during uniaxial compression or triaxial compression loading (See Figure 8-6). The equivalent elastic modulus is the slope of the F-, curve and can be assessed as either a tangent value (E =)F/) ,) or a secant value (E = F/,) from the initial loading. Also, it may be evaluated from an unload-reload cycle implemented off of the initial loading ramp. Most common in engineering practice, the tangent value taken at 50% of ultimate strength is reported as the characteristic elastic modulus (E_{R50}).

Intact rock specimens can exhibit a wide range of elastic modulus, as evidenced by Table 10-4. For these data, the measured values vary from 3.6 to 88.3 GPa (530 to 12815 ksi), with a mean value of E_R = 44.6 GPa (6500 ksi). Notably, these moduli are comparable to normal and high-strength concretes that are manufacturered for construction. For many sedimentary and foliated metamorphic rocks, the modulus of elasticity is generally greater parallel to the bedding or foliation planes than perpendicular to them, due to closure of parallel weakness planes.

An intact rock classification system based on strength and modulus ratio (E/F_u) is given in Table 10-5. For each of the basic rock types (igneous, sedmentary, and metamorphic), Figure 10-9 shows the corresponding groupings of elastic modulus (E_t) vs. uniaxial compressive strength (F_u). The modulus here is the tangent modulus at 50% of ultimate strength. The broad range of strengths and moduli shown in the three figures is informative. The above system considers intact rock specimens only and does not consider the natural fractures (discontinuities) in the rock mass.

TABLE 10-5

ENGINEERING CLASSIFICATION OF INTACT ROCK

(Deere and Miller, 1966; Stagg and Zienkiewicz, 1968)

I. On basis of strength, F_u

Class	Description	Uniaxial compressive strength (MPa)
A	Very high strength	Over 220
B	High strength	110-220
C	Medium strength	55-110
D	Low strength	28-55
E	Very low strength	Less than 28

II. On basis of modulus ratio, E_t/F_u

Class	Description	Modulus ratio [b]
H	High modulus ratio	Over 500
M	Average (medium) ratio	200-500
L	Low modulus ratio	Less than 200

[a] Rocks are classified by strength and modulus ratio such as AM, BL, BH, CM, etc.. [b]Modulus ratio = $E_t/F_{a(ult)}$ where E_t is tangent modulus at 50% ultimate strength and $F_{a(ult)}$ is the uniaxial compressive strength.

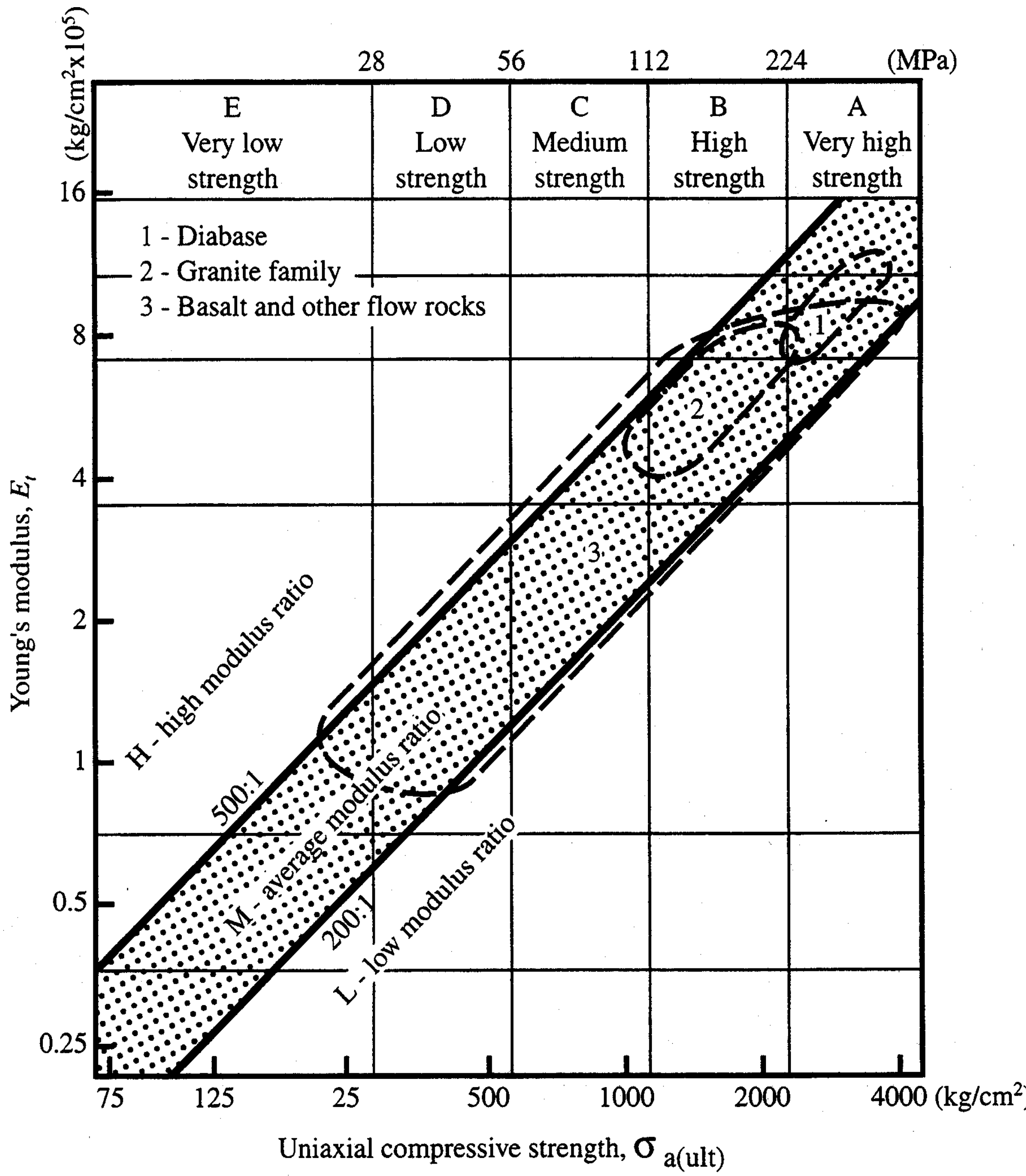

Figure 10-9a. Elastic Modulus-Compressive Strength Groupings for Intact Igneous Rock Materials (Deere & Miller, 1966).

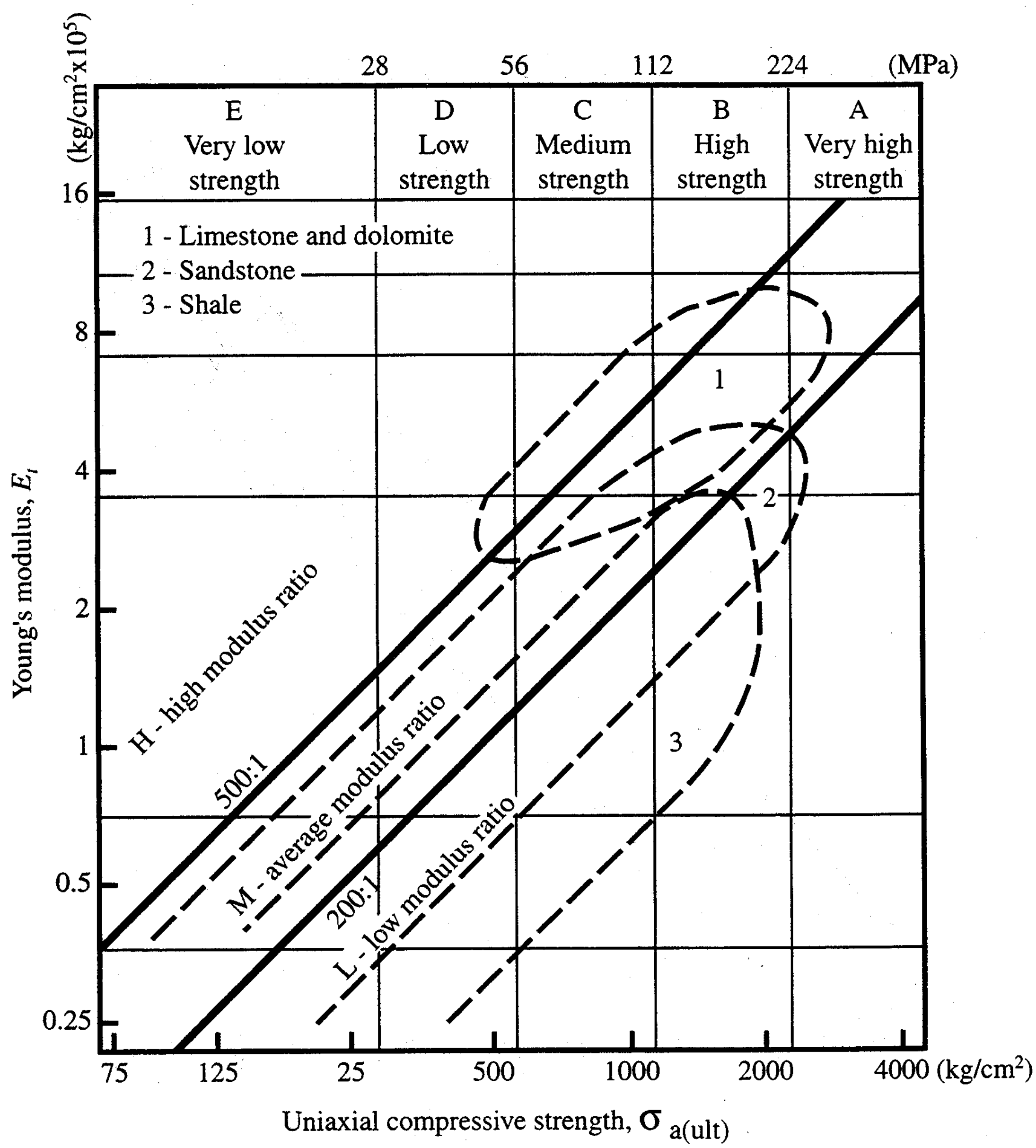

Figure 10-9b. Elastic Modulus-Compressive Strength Groupings for Intact Sedimentary Rock Materials (Deere & Miller, 1966).

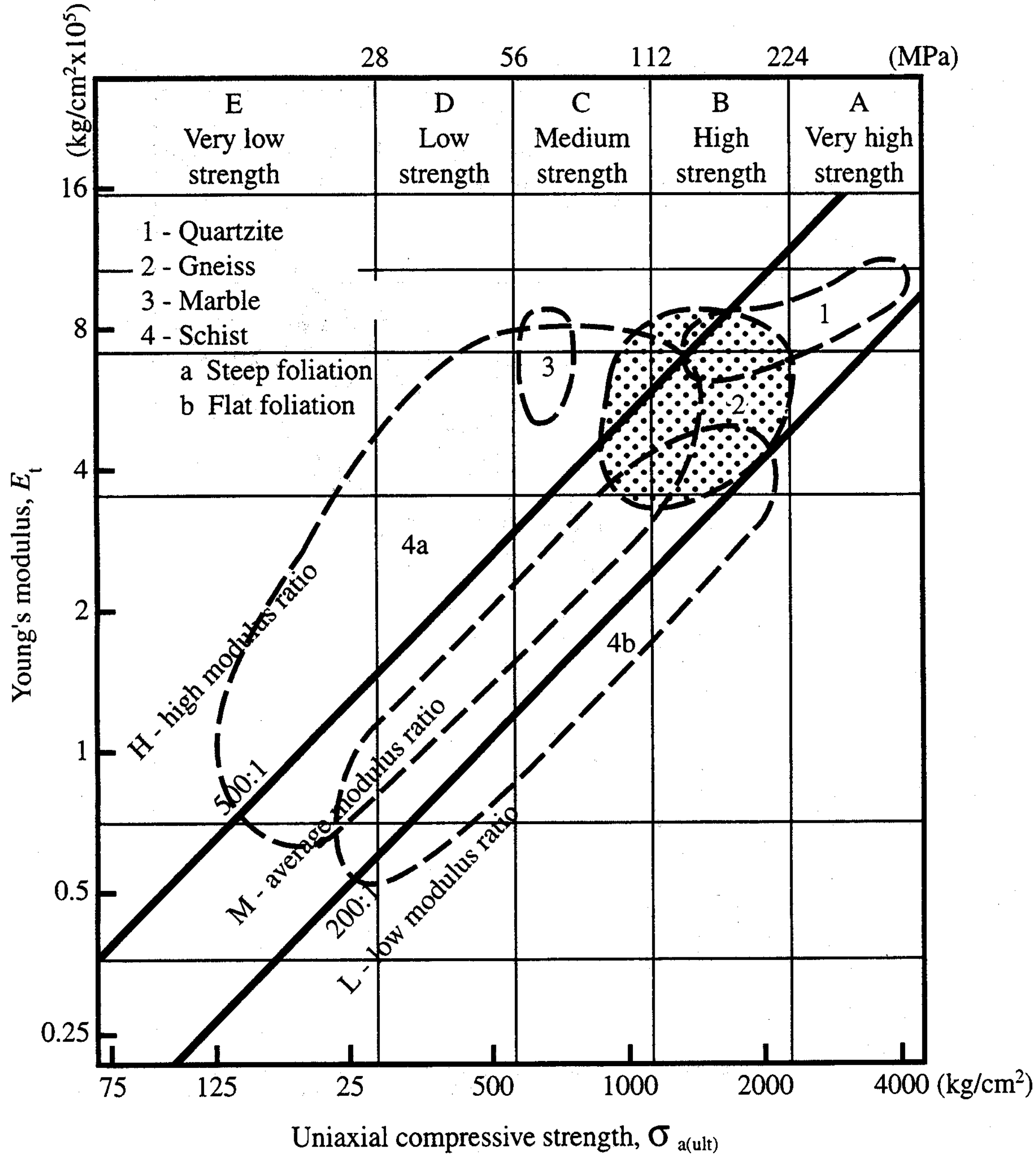

Figure 10-9c. Elastic Modulus-Compressive Strength Groupings for Intact Metamorphic Rock Materials (Deere & Miller, 1966).

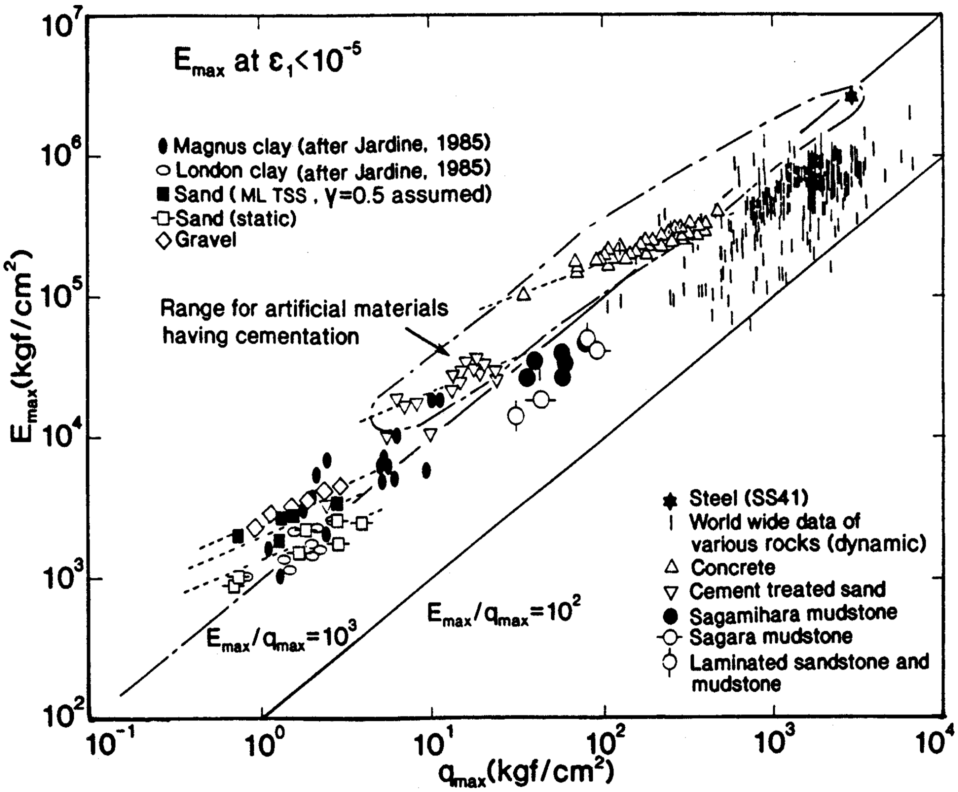

Figure 10-10. Small-Strain Elastic Modulus (E_{max}) versus Compressive Strength (q_u) for All Types of Civil Engineering Materials. (Tatsuoka & Shibuya, 1992).

For lab testing on intact rock specimens, the nondestructive elastic modulus at very small strains is obtained from ultrasonics measurements and this value is higher than moduli measured at intermediate to high strains, such as E_{t50}. Figure 10-3d shows a global database of E_{max} from small-strain measurements (ultrasonics, bender elements, resonant column) versus the compressive strength ($q_{max} = q_u$) for a wide range of civil engineering materials ranging from soils to rocks, as well as concrete and steel (Tatsuoka & Shibuya, 1992).

10.3 Operational Shear Strength

The shear strength of rock usually controls in the geotechnical evaluation of slopes, tunnels, excavations, and foundations. As such, the shear strength (τ) of inplace rock often needs to be defined at three distinct levels: (a) intact rock, (b) along a rock joint or discontinuity plane, and (c) representative of an entire fractured rock mass. Figure 10-11 illustrates these cases for the illustrative example involving a road highway cut in rock. In all cases, the shear strength is most commonly determined in terms of the Mohr-Coulomb criterion (Figure 10-7):

$$\tau = c' + \sigma' \tan \phi' \qquad (10\text{-}3)$$

where τ = operational shear strength, σ' = effective normal stress on the plane of shearing, c' = effective cohesion intercept, and ϕ' = effective friction angle. The appropriate values of the Mohr-Coulomb parameters c' and ϕ' will depend greatly upon the specific cases considered and levels of failure applicable per Figure 10-11.

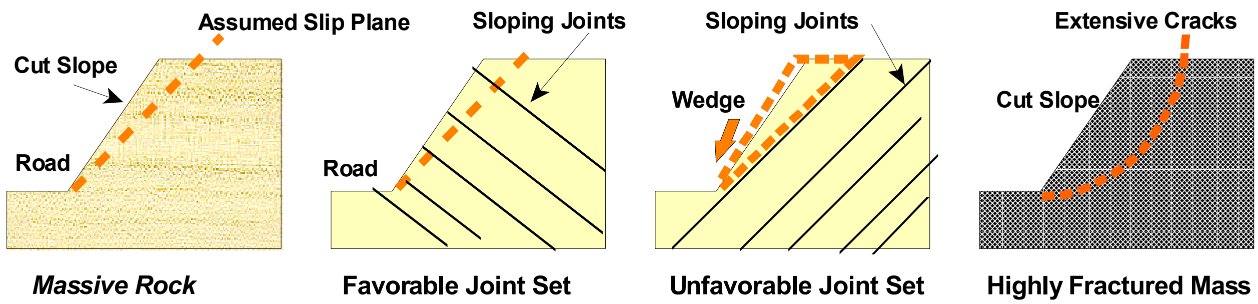

Figure 10-11. Illustrative Cases for Defining Rock Shear Strength for Cut, including: (a) intact rock strength, (b) intact strength across joints, (c) shear strength along joint planes, and (d) jointed rock mass.

For the intact rock, series of triaxial compressive strength tests can be performed at increasing confining stresses to define the Mohr-Coulomb envelope and corresponding c' and ϕ' parameters. See Section 7.1.8 for further details on this approach. Alternatively, empirical methods based on the type of rock material and its measured uniaxial compressive strength ($q_u = \sigma_u$) are available for evaluating the shear strength parameters of intact rock (e.g., Hoek, et al. 1995), as discussed later in Section 10.4. This approach is versatile as it can be reduced to account for the degree of fracturing and weathering, thus also used to represent and estimate the shear strength of rock masses.

Laboratory direct shear testing can be used to determine the shear strength of a discontinuity and/or the infilling material found within the joints. The split box is orientated with the axis along the preferred plane of interest (Figure 8-4). The shear strength of the discontinuity surface has either a representative peak or residual value of the frictional component of shear strength. Peak shear strengths will apply during highway cuts and excavations in rocks where no movement has occurred before. Residual shear strengths will be appropriate in restoration and remedial work involving rockslides and slipped wedges or blocks of rock. Relatively small movements can reduce shear strength from peak to residual values. The peak values can be conceived as the composite of the residual shear strength and a geometrical component that depends on roughness and related to asperities and roughness on the joint plane. Table 10-6 lists values of peak friction angle of various rock surface types, rock minerals (that may coat the joints), and infilling materials (such as clays and sands). If the joints are open enough, the infilling of clay/soil may dominate the shear strength behavior of the situation.

Movement reduces (or removes) the effect of the asperities, resulting in reduced shear strength. If sufficent movement occurs, the residual strength of the material is reached. Table 10-7 presents a selection of reported values of residual frictional angle (ϕ_r', assuming $c_r' = 0$) for various types of rock surfaces and minerals found in rock joints and discontinuities. These values can give an approximate guide in selecting interface and joint strengths.

Additional guidelines for the selection of Mohr-Coulomb parameters are given by Hoek, et al. (1995) and Wyllie (1999).

TABLE 10-6

FRICTION ANGLES FOR ROCK JOINTS, MINERALS, AND FILLINGS

(after compilations by Franklin & Dusseault, 1989, and Jaeger & Cook, 1977)

Condition/Case	**Friction Angle N' (deg)** **(c' = 0)**
Thick joint fillings:	
Smectite and montmorillonitic clays	5 - 10
Kaolinite	12 - 15
Illite	16 - 22
Chlorite	20 - 30
Quartzitic sand	33 - 40
Feldspathic sand	28 - 35
Minerals:	
Talc	9
Serpentine	16
Biotite (mica)	7
Muscovite (mica)	13
Calcite	8
Feldspar	24
Quartz	33
Rock joints:	
Crystalline limestone	42 - 49
Porous limestone	32 - 48
Chalk	30 - 41
Sandstone	24 - 35
Quartzite	23 - 44
Clay Shale	22 - 37
Bentonitic Shale	9 - 27
Granite	31 - 33
Dolerite	33 - 43
Schist	32 - 40
Marble	31 - 37
Gabbro	33
Gneiss	31 - 35

TABLE 10-7

RESIDUAL FRICTION ANGLES

(compilations after Barton, 1973, and Hoek & Bray, 1977)

Rock Type	Residual Friction Angle N_r (degrees), assuming c' = 0
Amphibolite	32
Basalt	31-38
Conglomerate	35
Chalk	30
Dolomite	27-31
Gneiss (schistose)	23-29
Granite (fine grain)	29-35
Limestone	33-40
Porphyry	31
Sandstone	25-35
Shale	27
Siltstone	27-31
Slate	25-30

Note: Lower value is generally given by tests on wet rock surfaces.

10.4 ROCK MASS CLASSIFICATION

While the mineral composition, age, and porosity determine the properties of the intact rock, the network of fractures, cracks, and joints govern the rock mass behavior in terms of available strength, stiffness, permeability, and performance. The pattern of discontinuities of the rock mass will be evident in the cored sections obtained during the site exploration studies, as well as in the exposed faces and rock outcrops in the topographic terrain. A selection of exposed rock types is presented in Figure 10-12 to illustrate the variations that occur in scenery due to the inherent fracture and joint patterns.

Measures of quantifying the degree, extent, and nature of the discontinuities is paramount in assessing the quality and condition of the rock mass. The rock quality designation (RQD, described in Figure 3-20) is a first-order assessment of the amount of natural jointing and fissuring in rock masses. The RQD has been used to approximately quantify the rock mass behavior, yet was developed four decades ago (Deere & Deere, 1989). Since then, more elaborate and quantitative methods of assessing the overall rock mass condition have been developed including the Geomechanics RMR-System (Bieniawski, 1989), based on mining experiences in South Africa, and the NGI-Q system (Barton, 1988), based on tunneling experiences in Norway. A closely related system to the RMR is the Geological Strength Index (GSI) that will is useful in assessing the strength of rock masses. These and other rock mass classifications systems are described in detail elsewhere and summarized in ASTM D 5878 (Classification of Rock Mass Systems). The influential factors that comprise the rock mass ratings will be briefly discussed here and presented in the context for the interpretation of rock mass properties need for design and analysis of slopes, tunnels, and foundations in rock formations.

Figure 10-12 (a). Limestone at I-75, TN

Figure 10-12 (b). Sandstone in Grand Canyon. AZ

Figure 10-12 (c). Basalt Beach, Kauai, HI

Figure 10-12 (d). Mica Schist near Hope, BC

Figure 10-12 (e). Gneiss at Sondestrom, Greenland.

Figure 10-12(f). Exposed Granite, Rio, Brazil

Figure 10-12. Selection of Exposed Rock Masses from Different Geologic Origins.

10.4.1 Rock Mass Rating System (RMR)

The Rock Mass Rating (RMR) rock classification system uses five basic parameters for classification and properties evaluation. A sixth parameter helps further assess issues of stability to specific problems. Originally intended for tunneling & mining applications, it has been extended for the design of cut slopes and foundations. The six parameters used to determine the RMR value are:

- Uniaxial compressive strength (q_u or σ_u)*.
- Rock Quality Designation (RQD)
- Spacing of discontinuities
- Condition of discontinuities
- Groundwater conditions
- Orientation of discontinuities

*Note: Value may be estimated from point load index (I_s).

The basic components of the RMR system is contained in Figure 10-13. The rating is obtained by summing the values assigned for the first five components. Later, an overall rating can be made by a final adjustment by consideration of the sixth component depending upon the intended project type (tunnel, slope, or foundation), however, this is less utilized in most routine applications. Thus, the RMR is determined as:

$$RMR = \sum_{i=1}^{5} (R_i) \qquad (10\text{-}4)$$

The RMR rating assigns a value of between 0 (very poor) to 100 (most excellent) for the rock mass. The RMR system has been modified over the years with additional details and variants given elsewhere (e.g., Bieniawski, 1989; Hoek, et al., 1995; Wyllie, 1999). Depending upon the dip and dip direction (or strike) of the natural discontinuities with respect to the proposed layout and orientation of the construction, then an additional factor may be added to adjust the RMR, ranging from favorable (R_6 = 0) to very unfavorable (-12 for tunnels, -25 for foundations, and -60 for slopes).

10.4.2. NGI - Q Rating

The Q Rating was developed for assessing rock masses for tunneling applications by the Norwegian Geotechnical Institute (Barton, et al. 1974) and relies on six parameters for evaluation:

- Rock Quality Designation (RQD)
- J_n is the number of discontinuity sets in the rock mass (joint sets).
- J_r represents the roughness of the interface within the discontinuities, fractures, and joints.
- J_a describes the condition, alterations, and infilling material with the joints and cracks.
- J_w provides an assessment on the inplace water conditions.
- SRF is a stress reduction factor related to the initial stress state and compactness.

The individual parameters are assigned values per the criteria given in Figure 10-14 and then a complete Q rating is obtained as follows:

$$Q = \left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right)\left(\frac{J_w}{SRF}\right) \qquad (10\text{-}5)$$

ROCK MASS RATING (RMR) *also CSIR System*

Geomechanics System - (Bieniawski, 1984, 1989)

Geomechanics Classification for Rock Masses		
CLASS	DESCRIPTION	RANGE of RMR
I	Very Good Rock	81 to 100
II	Good Rock	61 to 80
III	Fair Rock	41 to 60
IV	Poor Rock	21 to 40
V	Very Poor Rock	0 to 20

$$\mathrm{RMR} = \sum_{i=1}^{5} R_i$$

NOTE: Rock Mass Rating is obtained by summing the five index parameters to obtain an overal rating RMR. Adjustments for dip and orientation of discontinuities being favorable or unfavorable for specific cases of tunnels, slopes, & foundations can also be considered.

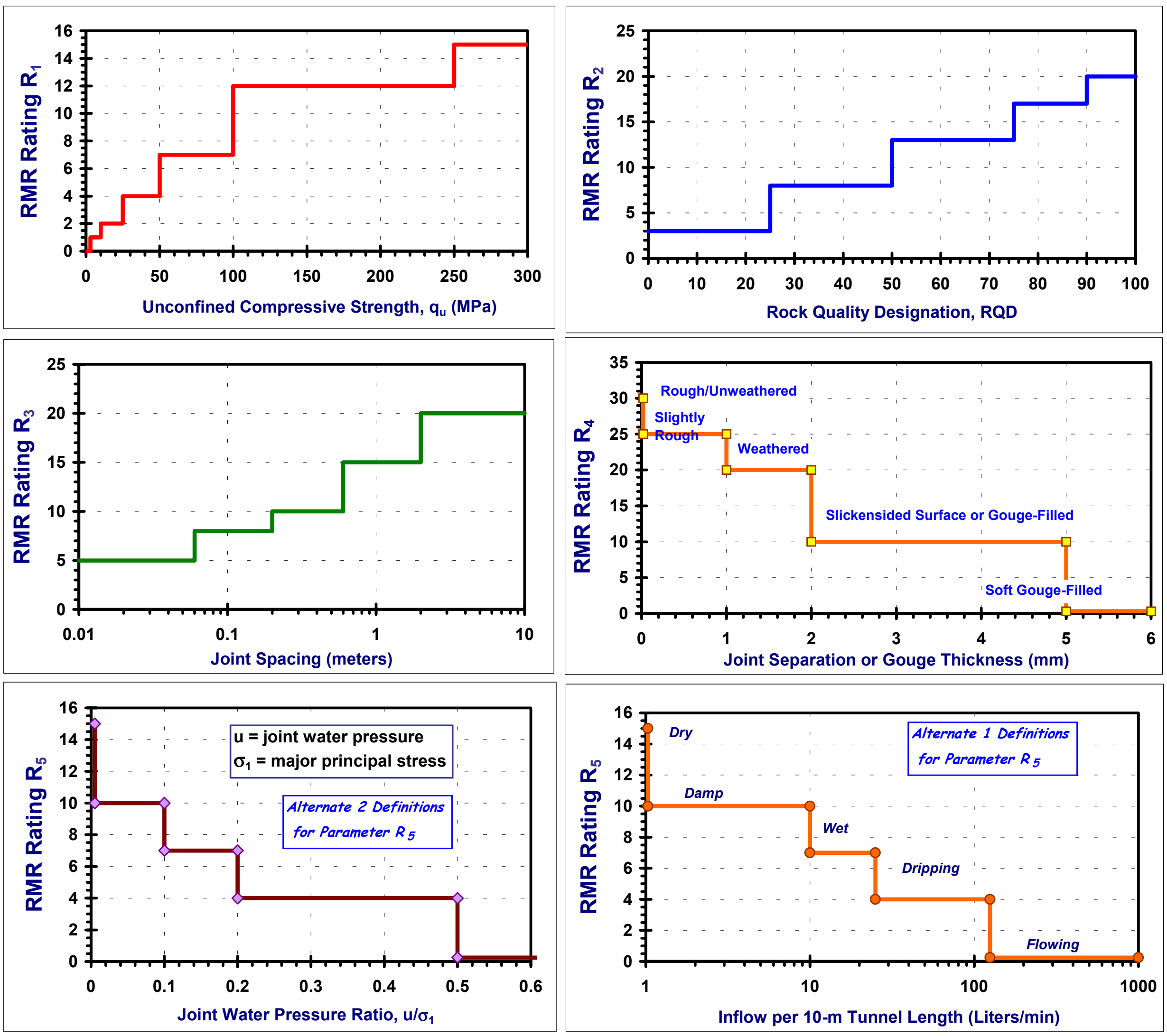

Figure 10-13. The Geomechanics Classification System for Rock Mass Rating (RMR)
(after Bieniawski, 1984, 1989).

NGI Q-System Rating for Rock Masses

(Barton, Lien, & Lunde, 1974)

$$Q = (RQD/J_n)(J_r/J_a)(J_w/SRF)$$

Norwegian Classification for Rock Masses

Q - Value	Quality of Rock Mass
< 0.01	Exceptionally Poor
0.01 to 0.1	Extremely Poor
0.1 to 1	Very Poor
1 to 4	Poor
4 to 10	Fair
10 to 40	Good
40 to 100	Very Good
100 to 400	Extremely Good
< 400	Exceptionally Good

PARAMETERS FOR THE Q-Rating of Rock Masses

1. RQD = Rock Quality Designation = sum of cored pieces > 100 mm long, divided by total core run length

2. Number of Sets of Discontinuities (joint sets)	= J_n
Massive	0.5
One set	2
Two sets	4
Three sets	9
Four or more sets	15
Crushed rock	20

3. Roughness of Discontinuities*	= J_r
Noncontinuous joints	4
Rough, wavy	3
Smooth, wavy	2
Rough, planar	1.5
Smooth, planar	1
Slick and planar	0.5
Filled discontinuities	1

*Note: add +1 if mean joint spacing > 3 m

4. Discontinuity Condition & Infilling	= J_a
4.1 Unfilled Cases	
Healed	0.75
Stained, no alteration	1
Silty or Sandy Coating	3
Clay coating	4
4.2 Filled Discontinuities	
Sand or crushed rock infill	4
Stiff clay infilling < 5 mm	6
Soft clay infill < 5 mm thick	8
Swelling clay < 5 mm	12
Stiff clay infill > 5 mm thick	10
Soft clay infill > 5 mm thick	15
Swelling clay > 5 mm	20

5. Water Conditions	
Dry	1
Medium Water Inflow	0.66
Large inflow in unfilled joints	0.5
Large inflow with filled joints that wash out	0.33
High transient flow	0.2 to 0.1
High continuous flow	0.1 to 0.05

6. Stress Reduction Factor**	= SRF
Loose rock with clay infill	10
Loose rock with open joints	5
Shallow rock with clay infill	2.5
Rock with unfilled joints	1

**Note: Additional SRF values given for rocks prone to bursting, squeezing and swelling by Barton et al. (1974)

Figure 10-14. The Q-Rating System for Rock Mass Classification
(after Barton, Lien, and Lunde, 1974).

Both the RMR and the Q-ratings can be used to evaluate the stand-up time of unsupported mine & tunnel walls which is valuable during construction. The RMR and Q are also used to determine the type and degree of tunnel support system required for long-term stability, including the use of shotcrete, mesh, lining, and rock bolt spacing. Details on these facets are given elsewhere (e.g., Hoek, et al., 1995).

10.4.3. Geological Strength Index (GSI)

Whereas the RMR and Q systems were developed originally for mining and tunnelling applications, the Geological Strength Index (GSI) provides a measure of the rock mass quality for directly assessing the strength and stiffness of intact and fractured rocks. A quick assessment of the GSI made be made by use of the graphical chart given in Figure 10-15, thus facilitating the procedure for field use.

More specifically, the GSI can be calculated from the components of the Q system, as follows:

$$GSI = 9 \cdot \log\left[\left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right)\right] + 44 \tag{10-6}$$

In relation to the common Geomechanics Classification System, the GSI is restricted to RMR values in excess of 25, thus:

For RMR > 25: $$GSI = \sum_{i=1}^{4} (R_i) + 10 \tag{10-7}$$

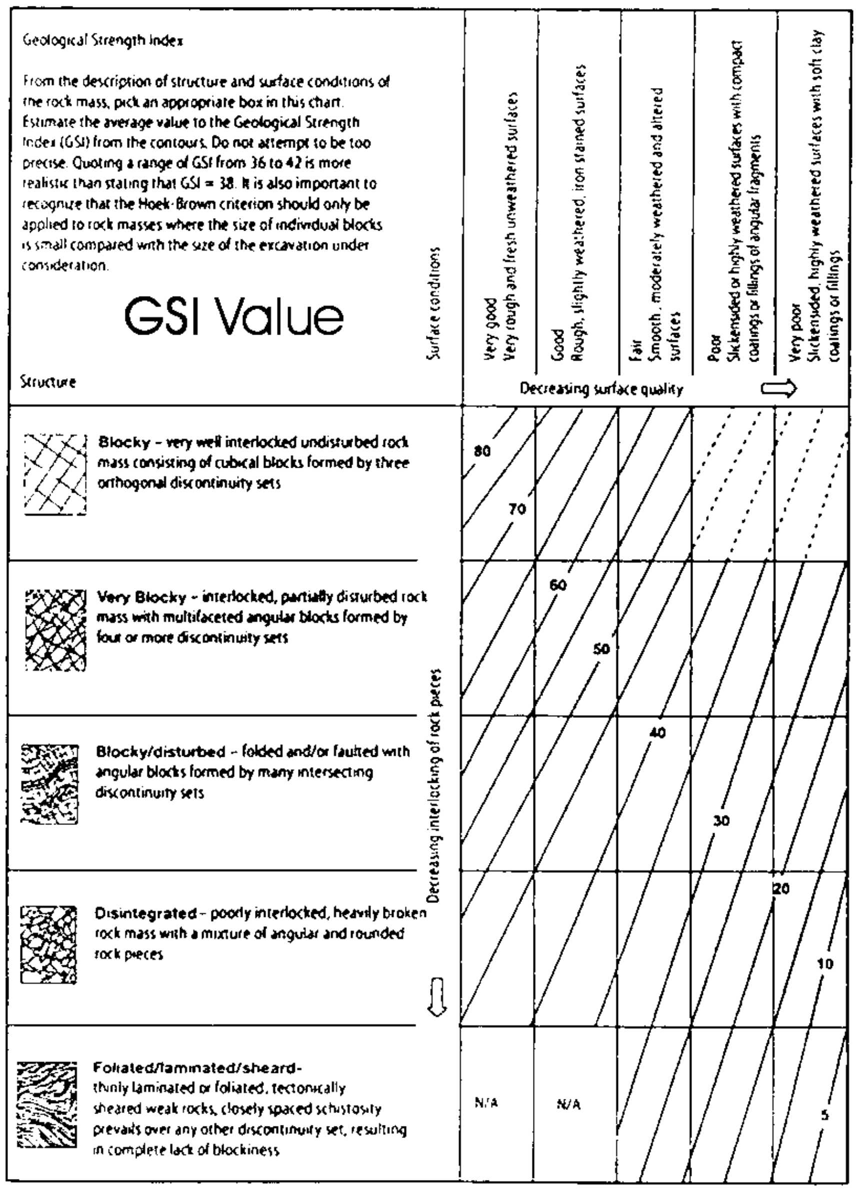

Figure 10-15. Chart for Estimating the Geological Strength Index (GSI) (Hoek & Brown, 1997).

10.5. ROCK MASS STRENGTH

The strength of the overall assemblage of rock blocks and fractures can be assessed by large direct shear tests conducted in the field, backcalculation of rockslides and failured slopes, or alternatively estimated on the basis of rock mass classification schemes. For the latter, a detailed approach to evaluating the rock mass strength is afforded through use of the GSI rating (Hoek, et al. 1995). In this method, the major principal stress (σ_1') is related to the minor principal stress (σ_3') at failure through an empirical expression that depends upon the following:

- The uniaxial compressive strength of the rock material (σ_u)
- A material constant (m_i) for the type of rock
- Three empirical parameters that reflect the degree of fracturing of the rock mass (m_b, s, and a).

The relationship accounts for curvature of the Mohr-Coulomb strength envelope and gives the expression for major principal stress in the form:

$$\sigma_1' = \sigma_3' + \sigma_u \left[m_b \frac{\sigma_3'}{\sigma_u} + s \right]^a \qquad (10\text{-}8)$$

The material parameter m_i depends on the spectific rock type (igneous, metamorphic, or sedimentary) as determined from the chart given in Figure 10-16. Values range as low as 4 for mudstone to as high as 33 for gneiss and granite.

For GSI > 25, the remaining strength parameters for undisturbed rock masses are:

$$m_b = m_i \exp\left[(GSI-100)/28\right] \qquad (10\text{-}9)$$

$$s = \exp\left[(GSI-100)/9\right] \qquad (10\text{-}10)$$

$$a = 0.5 \qquad (10\text{-}11)$$

For GSI < 25, the parameter selection is given by:

$$s = 0 \qquad (10\text{-}12)$$

$$a = 0.65 - (GSI/200) \qquad (10\text{-}13)$$

Thus, the evaluation is easily carried out using a spreadsheet with adopted values of effective confining stresses (σ_3') taken over the range of anticipated field overburden stresses to calculate corresponding values of effective major principal stress at failure (σ_1') by equation (10-8). Then, the paired values of σ_1' and σ_3' can be plotted [using either Mohrs Circles or q-p plots] to obtain the equivalent shear strength parameters, c' and ϕ'. Note that the method can also be applied to evaluate the strength of intact rock (GSI = 100), as well as fractured rock. For quick assessments, representative and average values of σ_3' have been used to derive approximate chart solutions for selecting normalized c'/σ_u and friction angle ϕ' directly from GSI and material constant m_i, as presented in Figure 10-17.

Rock type	Class	Group	Texture			
			Course	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerate (22)	Sandstone 19	Siltstone 9	Claystone 4
				← Greywacke (18) →		
	Non-Clastic	Organic		← Chalk 7 →		
				← Coal (8-21) →		
		Carbonate	Breccia (20)	Sparitic Limestone (10)	Micritic Limestone 8	
		Chemical		Gypstone 16	Anhydrire 13	
METAMORPHIC	Non Foliated		Marble 9	Hornfels (19)	Quartzite 24	
	Slightly foliated		Migmatite (30)	Amphibolite 31	Mylonites (6)	
	Foliated*		Gneiss 33	Schists (10)	Phyllites (10)	Slate 9
IGNEOUS	Light		Granite 33		Rhyolite (16)	Obsidian (19)
			Granodiorite (30)		Dacite (17)	
	Dark		Diorite (28)		Andesite 19	
			Gabbro 27	Dolerite (19)	Basalt (17)	
			Norite 22			
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

Figure 10-16. Material Constant m_i for GSI Evaluation of Rock Mass Strength
(Hoek, et al., 1995).

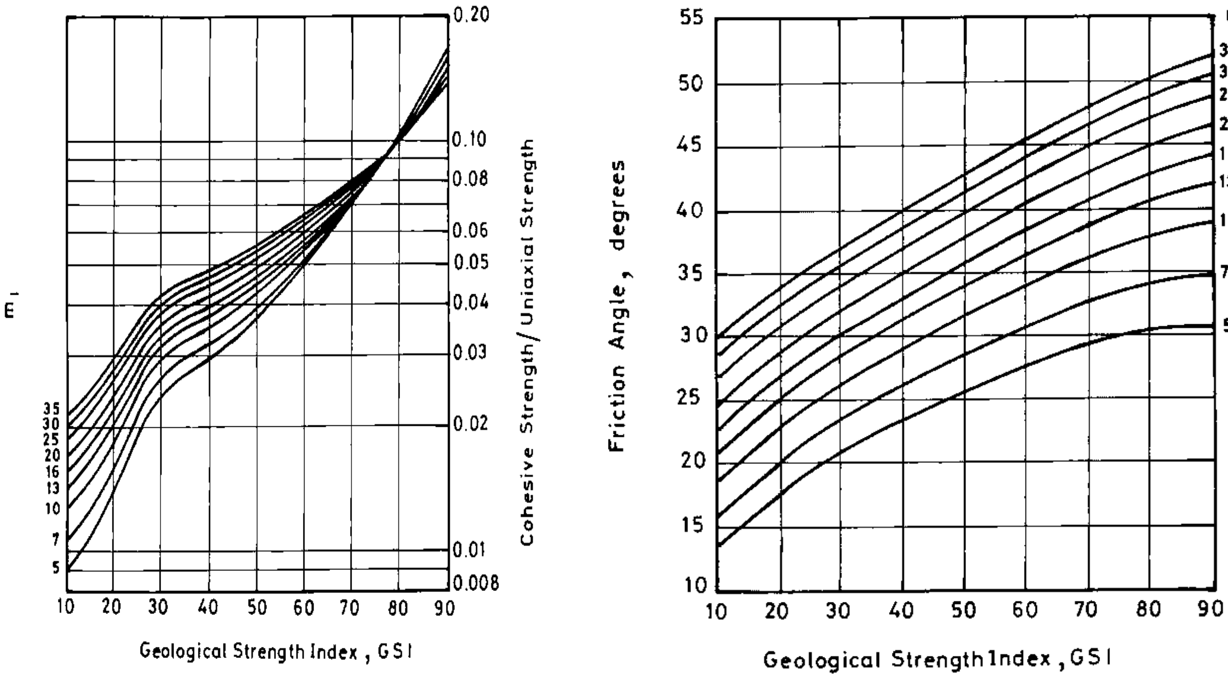

Figure 10-17. Approximate Chart Solution for Obtaining Normalized Cohesion Intercept (cr/F_u) and Friction Angle (Nr) from GSI Rating and m_i Parameter (After Hoek & Brown, 1997).

For the apparent shear strength along specific joints and planes of sliding, the peak friction angle can be evaluated from the Q-rating parameters (c' = 0):

$$\phi_p' \quad . \quad (J_r/J_a) \qquad (10\text{-}14)$$

which gives a range of 7° < ϕ_p' < 75° for the full value limits of joint roughness (J_r) and alteration (J_a) parameters.

10.6. ROCK MASS MODULUS

The equivalent elastic modulus (E_M) of rock masses is used in deformation analyses amd numerical simulations involving tunnels, slopes, and foundations to estimate magnitudes of movements and deflections caused by new loading. Field methods of measuring the deformability characteristics of rock masses include the Goodman jack and rock dilatometer, as well as backcalculation from full-scale foundation load tests (e.g., Littlechild, et al., 2000). For routine calculations, E_M has been empirically related to intact rock properties (uniaxial strength, F_u, and elastic modulus of the intact rock, E_R), rock quality (RQD), and rock mass ratings (RMR, Q, and GSI), such as given by the expressions listed in Table 10-7. On critical projects, the actual stiffness of the rock formation can be assessed using full-scale load tests, made more practical in recent times by the advent of the Osterberg load cell which can apply very large forces using embedded hydraulic systems.

TABLE 10-8

EMPIRICAL METHODS FOR EVALUATING ELASTIC MODULUS (E_M) OF ROCK MASSES

Expression	Notes/Remarks	Reference
For RQD < 70: $E_M = E_R$ (RQD/350) For RQD > 70: $E_M = E_R$ [0.2 + (RQD-70)37.5]	Reduction factor on intact rock modulus	Bieniawski (1978)
E_M . E_R [0.1 + RMR/(1150 - 11.4 RMR}]	Reduction factor	Kulhawy (1978)
E_M (GPa) = 2 RMR - 100	45 < RMR < 90	Bieniawski (1984)
E_M (GPa) = 25 Log_{10} Q	1 < Q < 400	Hoek et al. (1995)
E_M (GPa) = $10^{[RMR-100]/40}$	0 < RMR < 90	Serafim & Pereira (1983)
E_M (GPa) = $(0.01F_u)\ 10^{[GSI-100]/40}$	Adjustment for rocks with F_u < 100 MPa	Hoek (1999)

Notes: E_R = intact rock modulus, E_M = equivalent rock mass modulus, RQD = rock quality designation, RMR = rock mass rating, Q = NGI rating of rock mass, GSI = geologic strength index, F_u = uniaxial compressive strength.

10.7. FOUNDATION RESISTANCES

In many highway projects, foundations can bear on the rock surface or be embedded into the rock formation to resist large axial loads. For bridge structures, shallow spread footing foundations not subjected to scour can bear directly on the rock. In other instances, deep foundations may consist of large drilled shafts or piers that are constructed into the rock using coring methods. These may be designed for axial compression and/or uplift. In the following sections, methods of estimating the bearing stresses and side resistance in rocks are provided.

10.7.1 Allowable Foundation Bearing Stress

Detailed calculations can be made concerning the bearing capacity of foundations situated on fractured rock (e.g., Goodman, 1989). In addition, the results of the field and laboratory characterization program of the rock mass may be used to estimate the allowable bearing values directly. In the most simple approach, presumptive values are obtained from local practice, Uniform and BOCA building codes, and AASHTO guidelines. A summary of allowable bearing stresses from codes has been compiled by Wyllie (1999) and presented in Figure 10-18. If the RQD < 90%, the values given in the figure should be decreased by variable reduction factors ranging from 0.7 to 0.1. In this regard, the approach of Peck, et al. (1974) uses the RQD directly to assess the allowable bearing stress ($q_{allowable}$), provided that the applied stress does not exceed the uniaxial compressive strength of the intact rock ($q_{allowable} < F_u$). The RQD relationship is shown in Figure 10-19. For more specific calculations and detailed evaluations, the results of the equivalent Mohr-Coulomb parameters from either the GSI approach may be used in traditional bearing capacity equations, as discussed by Wyllie (1999).

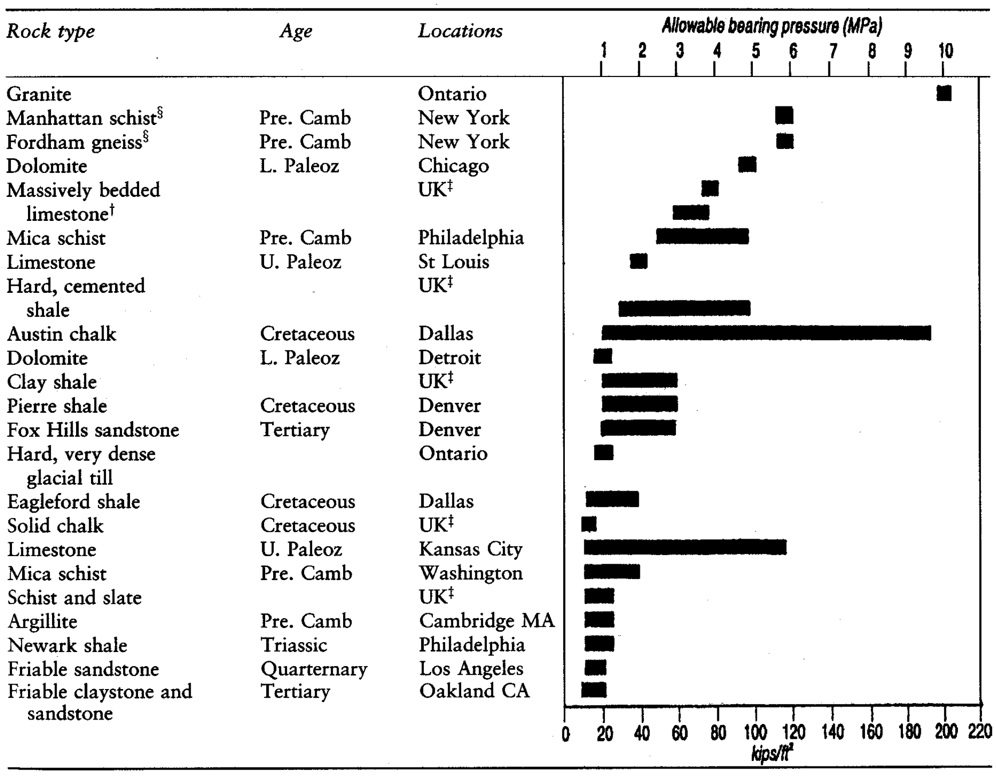

Figure 10-18. Allowable Bearing Stresses on Unweathered Rock from Codes (Wyllie, 1999).

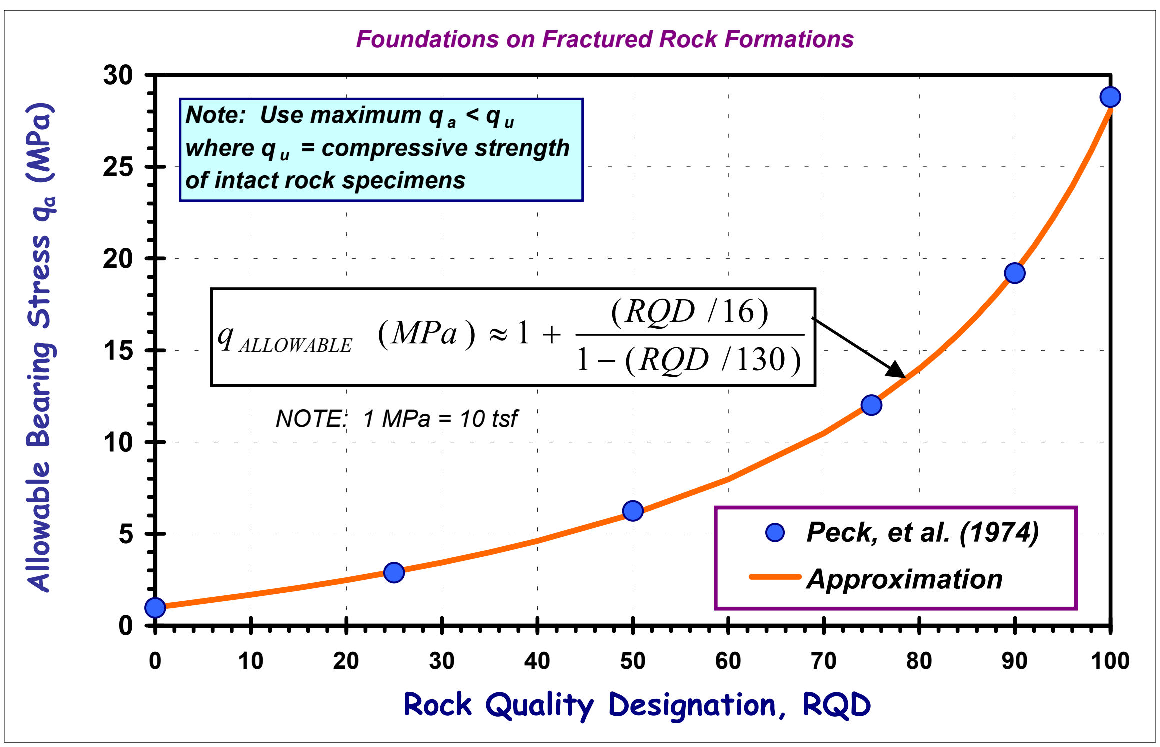

Figure 10-19. Allowable Bearing Stress on Fractured Rock from RQD (after Peck, et al. 1974).

10.7.2. Foundation Side Resistances

Deep foundations can be constructed to bear within rock formations.to avert scour problems and resist both axial compression and uplift loading. Drilled shaft foundations can be bored through soil layers and extended deeper by coring into the underlying bedrock. In many cases, the diameter of the drilled shaft is reduced when penetrating the rock, thus making a socket. Figures 10-20 presents a relationship between the shaft side resistance (f_s) and one-half the compressive strength ($q_u/2$) for sedimentary rocks, while Figure 10-21 shows a similar diagram between f_s and q_u for all rock types.

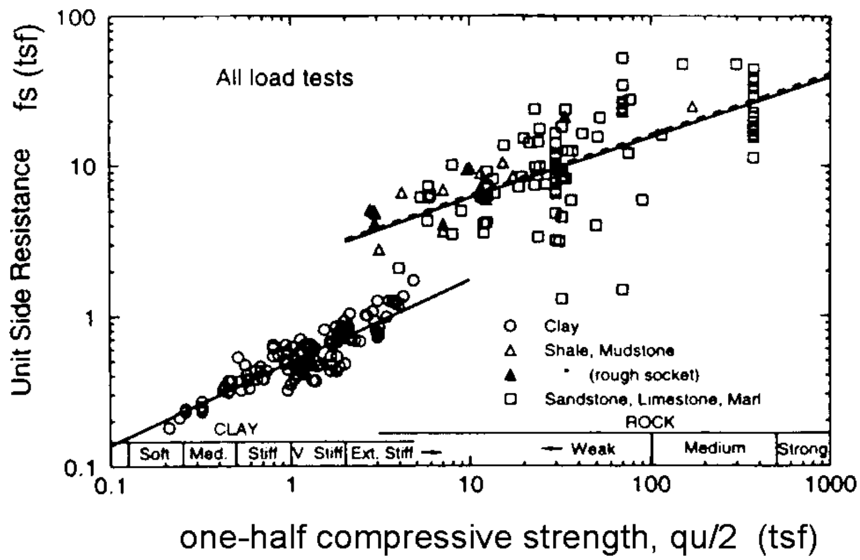

Figure 10-20. Unit Side Resistance Trend with Strength of Sedimentary Rocks (Kulhawy & Phoon, 1993).

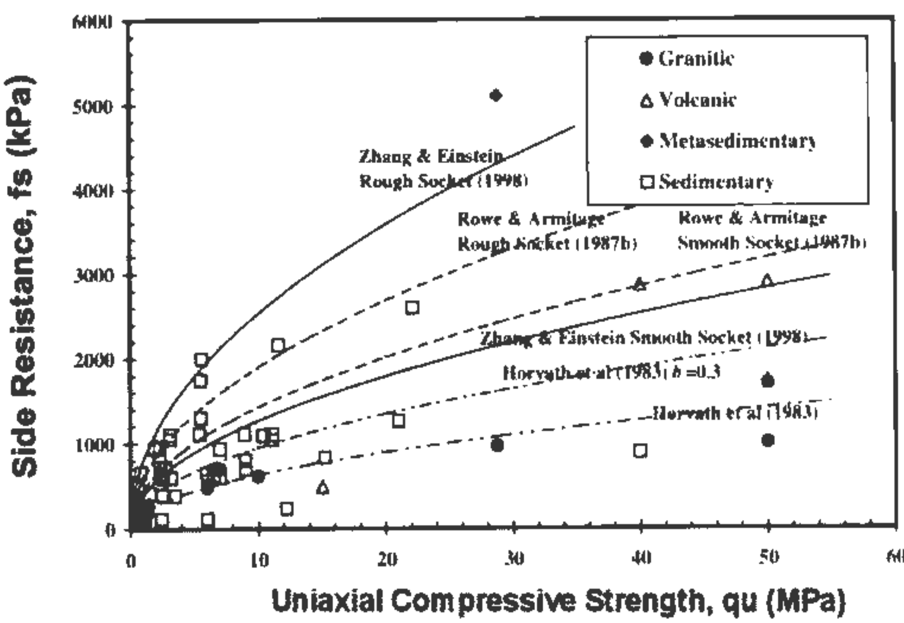

Figure 10-21. Shaft Unit Side Resistance with Various Rock Types (From Ng, et al., 2001).

10-8. Additional Rock Mass Parameters

As projects become more complex, there is need to measure and interprete additional geomechanical properties of the intact rock and rock mass. Some recent efforts have included assessments of scour and erodibility that have been related to rock mass indices (Van Schalkwyk, et al., 1995). Similar methodologies have been developed for excavatability of rocks by machinery in order to minimize use of blasting (Wyllie, 1999). A simple approach for the latter purpose utilizes the compression wave velocity (V_P) of the inplace rock directly, as shown in Figure 10-22.

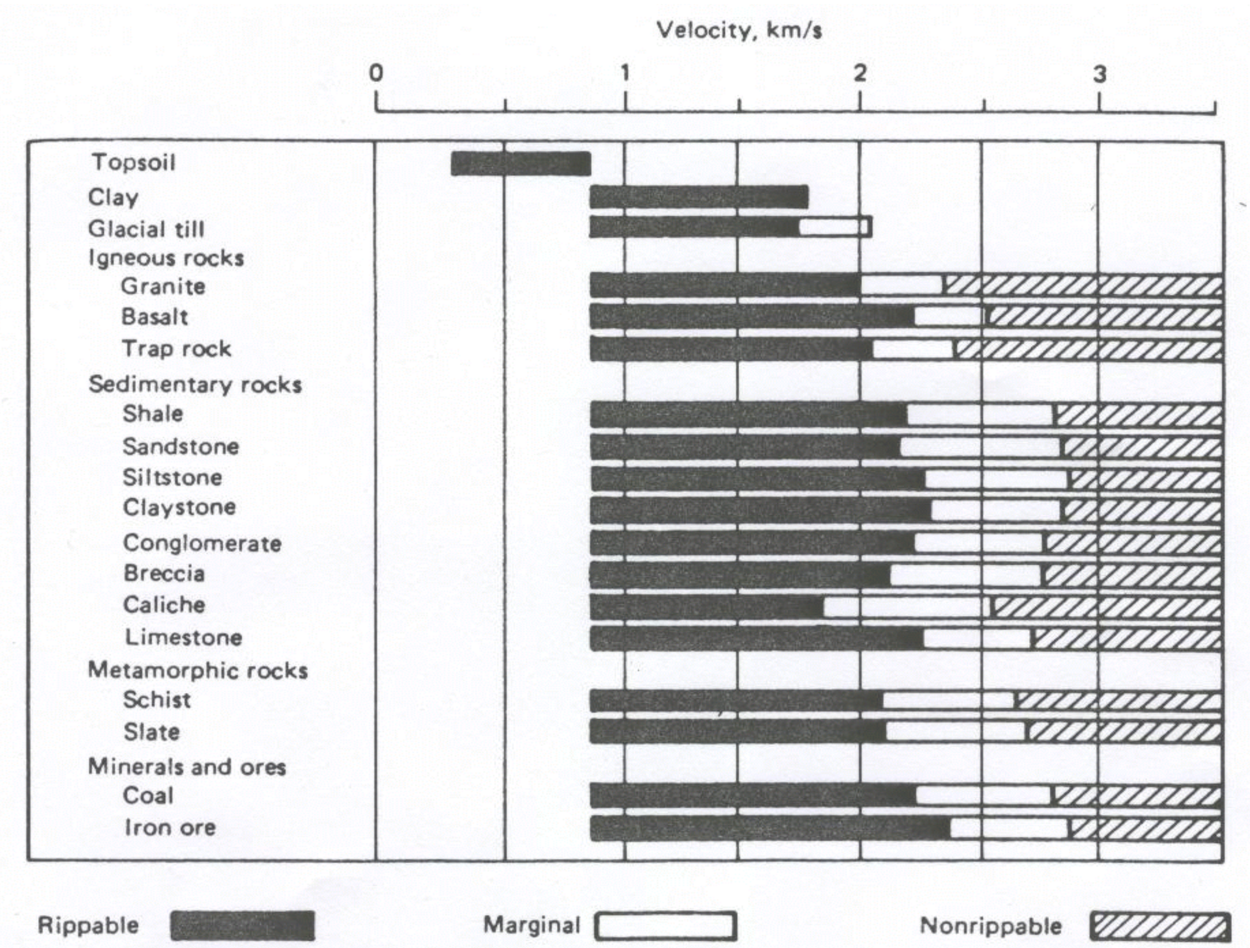

Figure 10-22. Rippability of Inplace Rock by Caterpillar Dozer Evaluate by P-Wave Velocity.
(After Franklin and Dusseault, 1989)